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Air-Regulated Siphon Spillways: Performance, Modeling, Design, and Construction

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AIR-REGULATED SIPHON SPILLWAYS:

PERFORMANCE, MODELING, DESIGN, AND CONSTRUCTION

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ABSTRACTS

Chapter 2

Little data exists in the literature for quantification of siphon spillway performance. Proper design of an air regulated siphon spillway requires knowledge of required flow rates and minimum vent size. A set of small siphon spillways were constructed to measure flow rate and required vent size relative to physical characteristics including pipe diameter, length of pipe, and elevation head. Vent sizing was shown to be related to the natural log of flow rate. Results were used to develop predictive models for flow rate and vent sizing. Models were validated and refined through testing on a siphon spillway installed on a pond at LaMaster Dairy Farm in Clemson, South Carolina.

Chapter 3

A common problem in pond performance is deterioration and reduced functionality of the primary spillway. Many water control structures are nearing the end of their materials' life expectancy. Primary spillways constructed of corrugated metal pipe have a life expectancy of thirty years (Montana DNR, 2012). The objective of this project is 1. To develop a set of design and construction guidelines for pond spillway rehabilitation using an air-regulated siphon spillway system. This type of water control structure will be useful to landowners as an economical and reliable replacement for vertical riser spillways.

To examine the application of the guidelines outlined in this chapter, a failed 40 year old water control structure at LaMaster Dairy Farm on the campus of Clemson University was replaced with a siphon spillway. The existing riser pipe rusted off approximately 1 m (3 ft) below

its normal design elevation, resulting in an estimated loss of 1.5 ha m (16 ac ft) of storage. Using a 2 year, 24 hour peak runoff rate from the 47.8 ha (118 ac) pond drainage area, calculations showed an 8-inch siphon would be conservatively sufficient. As part of the guidelines developed here, Natural Resources Conservation Service (NRCS) specifications and standards are used in curve number estimation, siphon sizing and elevation, runoff calculations and emergency spillway design. Construction procedures are also reviewed in this paper. The guidelines are a tool to show the correct steps in designing and constructing a siphon spillway.

Acknowledgments

I would like to thank Dr. Kirk and Clemson University for the opportunity to pursue a higher education. I would like to thank the Certified Erosion Prevention and Sediment Control Inspector (CEPSCI) program for funding my research. Clemson University Research Farms Service should also be mentioned for allowing me to use their pond and space for research. I would like to thank the undergraduates that helped design and install the LaMaster siphon system: Chris Bouknight, Matt Linder, Mike Parker, and Dan Ulmer. I'd also like to thank Guy Ramsey for his help during testing the LaMaster Siphon. Jacob Fravel and Dr. Kirk were tremendous help during the fabrication and calibration of the trailer scale. I am very thankful for every contribution to this project, large and small.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF FIGURES CONTINUED

LIST OF TABLES

CHAPTER 1

INTRODUCTION

Ponds are useful fixtures in the landscape. Ponds have been used for sediment control, livestock and crop irrigation, water supplies, and wetlands protection among other purposes. As they became a more common feature on a farmstead, several different types of water control structures or spillways, were developed providing more options for a landowner to use on his/her pond. A problem that has increased for ponds with corrugated metal pipe used in spillway construction is that the material was only designed to last 25 to 30 years (Montana DNR, 2012). Pipe degradation has led to the water control structures being compromised resulting in loss of control of the water surface elevation. A common occurrence is that the riser will deteriorate below the designed water elevation resulting in the pond being unable to maintain designed full pool. To remedy this problem, the pond spillway must generally be completely reconstructed or an alternative solution must be implemented.

Several terms are important in the discussion of ponds and pond design. Flood storage is the volume between principle spillway elevation and emergency spillway elevation, which represents the volume of water which can be stored from a rainfall or runoff event. Freeboard is the elevation difference between the emergency spillway and the top of dam. Freeboard is a safety factor that protects water from overtopping the dam. Normal pool elevation and principle spillway elevation are the same elevation. Pond storage volume is generally used to indicate the water stored below normal pool elevation. Figure 1.1 illustrates these terms.

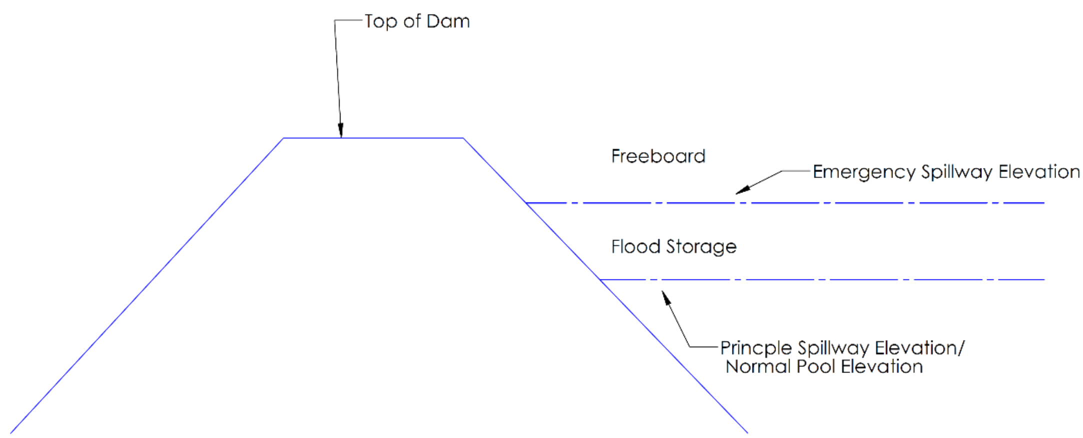

Figure 1.1. Pond Terminology Figure. Illustrates terms used frequently in relation to ponds and pond design.

When pond construction was at its peak during the 60's and 70's, the price of energy, especially fuel, was less than today. Diesel fuel has increased 346% from 1995 to 2011 (EIA, 2012) The rise in fuel prices have driven construction rates up, limiting the ability of some landowners to install traditional risers as water control structures for ponds. There are two practical alternative solutions to installing a traditional riser. One alternative solution is to allow the emergency spillway to function as the primary spillway. This method results in an increase in maintenance on the emergency spillway, as regular or continuous discharge will likely accelerate emergency spillway erosion. Allowing the emergency spillway to function as the primary spillway may also result in a loss of part of the fish population during large storm events. This would be the most economical alternative in the short term for the landowner, but would incur increased maintenance costs over the long term from erosion of the channel. The other water control structure alternative solution is the siphon spillway system. An advantage of installing a siphon spillway is the reduced earthwork when compared to reinstalling a vertical riser system and ability to install without complete pond drainage. Since the most common material used for siphon spillway is PVC, the siphon spillway materials could last up to 50 years (Pvc.org) years with proper maintenance. Siphon spillways are a viable alternative for pond rehabilitation.

An air-regulated siphon is broken into four sections (Fig. 1.2.): Inlet, Vent, Barrel, and Outlet. The inlet section is situated along the front slope of the dam and allows water to move into the system. The vent allows air entry providing a way for the system to be self-regulating, with an automatic startup and shut-off. The barrel of the system is where the pipe passes through the dam. Normal water elevation is the same elevation as the vent opening elevation and the invert elevation of the barrel, which is generally also the same elevation of the vent opening. The outlet of the pipe is situated along the backslope of the dam and discharges the water that has passed through the system. Terms used to describe siphon spillways such as those discussed in this paper include: air regulated siphon spillway, standard siphon spillway, gravity siphon and self-priming siphon. In this paper, all of these terms are synonymous with the siphon spillway design shown in Figure 1.2.

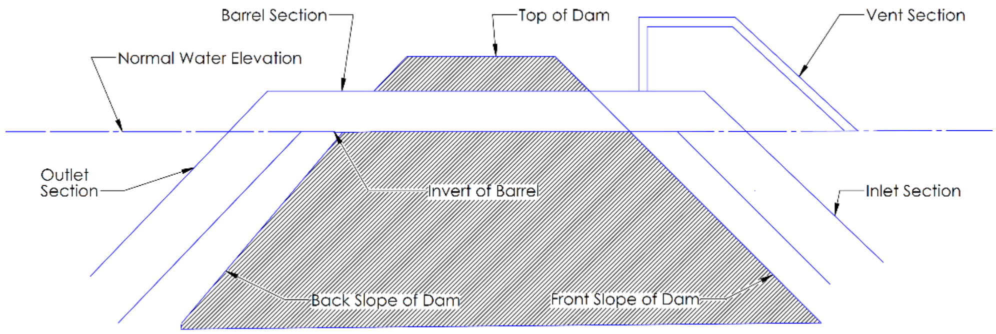

Figure 1.2. Sections of Siphon Figure. Details the sections of a siphon spillway and their relationship to normal water level.

There exists little literature characterizing siphon spillway performance as a function of physical characteristics. The projects presented here outline models for quantification of siphon flow rates, vent sizing and guidelines for design and construction of a siphon spillway. A budget analysis from a siphon spillway installed at Clemson University is also included for comparison to conventional spillway replacement. The analysis shows the cost of the siphon spillway compared to the cost of reconstructing the vertical riser system that was in originally in place. This collection of information should help siphon spillways become a practical, viable alternative for pond rehabilitation projects.

LITERATURE REVIEW

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CHAPTER 2

SIPHON SPILLWAY PERFORMANCE AND MODELING: FLOW AND VENT SIZING

ABSTRACT

Little data exists in the literature for quantification of siphon spillway performance. Proper design of an air regulated siphon spillway requires knowledge of required flow rates and minimum vent size. A set of small siphon spillways were constructed to measure flow rate and required vent size relative to physical characteristics including pipe diameter, length of pipe, and elevation head. Vent sizing was shown to be related to the natural log of flow rate. Results were used to develop predictive models for flow rate and vent sizing. Models were validated and refined through testing on a siphon spillway installed on a pond at LaMaster Dairy Farm in Clemson, South Carolina.

INTRODUCTION

Pond rehabilitation has become a common occurrence. Rising fuel prices have made earthwork an expensive part of projects. A common method of rehabilitation that reduces the volume of earthwork is installing a siphon spillway to control water level in a water-body. Siphons have been used since at least 1909 in the United States as water control structures (Gramatky, 1928). A typical siphon spillway pipe diameter is 150 mm (6 in) or 200 mm (8 in), while they can be built as large as the Karnataka in Hirebhasgar, India, which is 5.5 m (18 ft) in diameter (Thandaveswara, 2006). Correctly installed siphons can maintain water level by self-priming and automatic shutdown. Siphon spillways are self-priming and remove air from the system by the natural flow of water into the structure as water level in the upstream waterbody rises (Ahmadi, 2002).

Babaeyan-Koopaei et al. (2002) provided a detailed explanation of siphon spillway operating stages which are summarized here; an air regulated siphon

There are four stages to siphon flow: non-pressurized weir flow, subatmospheric weir flow, partial flow, and blackwater flow (Babeyan-Koopaei et al., 2002). Each of these stages has a unique relationship between air entrapment, entrainment, and full pipe flow. Air entrapment describes when a bubble of air is taking up space in pipe. This prevents the system from reaching full pipe flow and reduces momentum for the system. Air entrainment describes air that is moving through the system and becoming part of the discharge. A result of this physical phenomena in siphon systems is a "whitewater" visual effect at the outlet. This is a reduced discharge rate from the full pipe flow.. Full pipe flow is exhibited as fluid flowing through the entire cross sectional area of the pipe. When the pond water level is discharged below the elevation of the vent, the siphon operating stages should go from full pipe flow to air entrainment to air entrapment.

1. Non-pressurized weir flow is when water level has risen to an elevation higher than invert of barrel inlet but not high enough to charge siphon. The flow is still at atmospheric conditions as water has not completely sealed the barrel inlet. This is demonstrated as a trickle flow or gravity flow. The flow rate in this stage will be considerably less than the other stages of flow.
2. Subatmospheric weir flow occurs when the water level has risen to an elevation higher than the top of the barrel inlet and seals the inlet. Once the barrel inlet has been sealed by the rising water, air begins to exit the system causing a vacuum to

occur. The increase of falling water seals the outlet of the siphon momentarily, which increases the vacuum in the system as the volume of discharge increases. Subatmospheric weir flow demonstrates a higher volume of discharge compared to non-pressurized weir flow.

3. Partial flow begins when there is air and water exiting the siphon. A pulsed pattern of air and water will continue until all entrapped and entrained air exits the system. In this stage, siphon is beginning to stabilize and seal the outlet continually, allowing no air to enter the system. Flow will be sporadic and strong during this period.
4. Blackwater flow occurs when the siphon conduit is in full flow with no air in the system. This occurs when the system is operating at full discharge capabilities. Siphons with low elevation heads may not demonstrate blackwater flow. When this occurs, it is due to the water level not being raised over the barrel and vent sufficiently.

As water level decreases, the vent on an air-regulated siphon spillway will be exposed to air. This air being entrained through the vent de-primes the siphon, causing water flow to dissipate. As the length of pipe on the outlet increases, the additional pipe increases a tendency for a vacuum to occur that requires a larger vent to induce shutdown. (Babeyan-Koopaei et al., 2002).

Very little information was found in academic literature about the performance of siphon spillways. Most academic research found on siphon spillways pertained to a rectangular cross sectional siphon spillway (Tadayon, 2013; Ghafourian, 2012; and Gramatky, 1928) rather than a circular PVC based siphon spillway. A goal of this project was to build a predictive model based upon experimental data from small siphons to estimate flow for field siphon spillways. Such a model could be used by landowners to design and build siphon systems to regulate water level within deteriorating pond systems. Design specifications to be used in this predictive model include: 1. Pipe diameter, 2. Length of pipe, and 3. Elevation head. As need for pond rehabilitation projects increase with aging infrastructure, the public will need a tool to help design a siphon system. There are specifications and standards that NRCS has to install siphon spillways.

A goal of this project was to determine how to correctly size vent holes in siphon systems. There have been cases in South Carolina of a siphon not de-priming and continuing to discharge water until the pond was dry (Eddie Martin, USDA-NRCS Engineer, Interview, 29 March 2013). A goal of this research is to determine if a 50 mm (2 in) vent diameter is insufficient for 200 mm (8 in) siphon spillways or if there was another reason that the siphon failed to de-prime.

An issue with the vent in siphon spillways is the energy present when de-priming occurs. When a siphon spillway drains a water body below the vent level, the sudden change in flow and pressure entering and exiting the system will stress the pipe and cause flexing to occur. This could cause an increase in maintenance issues for the structure in the long term in an air-regulated siphon. A properly designed siphon spillway introduces air into the siphonic cycle to ensure a smooth, gradual, controlled action when priming and de-priming (Ahmadi, 2002).

OBJECTIVES

1. Quantify siphon flow rates and minimum vent diameters for experimental siphon spillways.
2. Develop a predictive model for siphon spillway flow rate as a function of hydraulic characteristics.
3. Develop a predictive model for minimum required siphon spillway vent diameter as a function of flow rate and hydraulic characteristics.
4. Validate and refine predictive siphon flow rate and vent sizing models using data from LaMaster siphon spillway.

MATERIALS AND METHODS

Three experimental siphon spillways were constructed, each of which were tested at five head differentials, as seen in Figure 2.1. Each of the five head differentials for the three spillways were also tested at two different inlet section pipe lengths. The siphons were constructed of Schedule 40 PVC with diameters as follows: 25 mm (1 in), 37.5 mm (1.5 in) and 50 mm (2in).

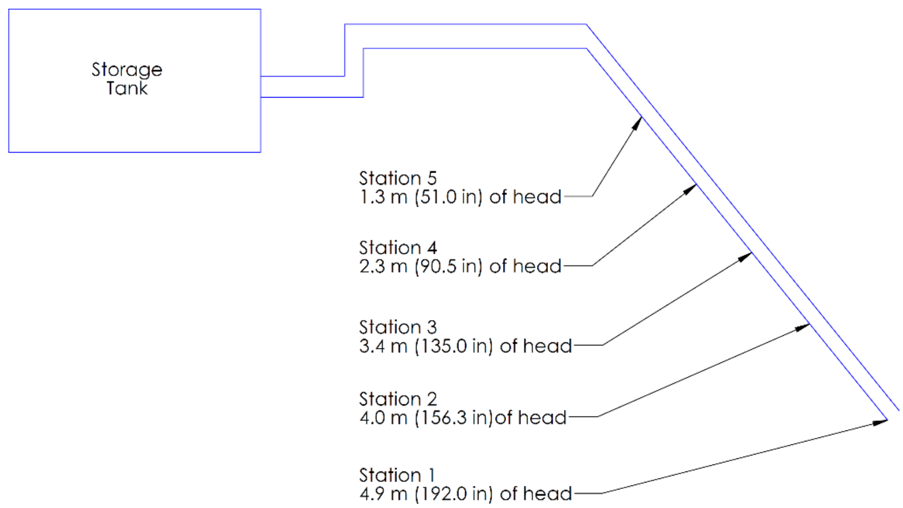

Figure 2.1: Small Siphon Experiment Stations Figure. Flow observations and Vent observations were recorded at each station.

The discharge section of the siphons were constructed by using removable sections made by cutting a piece of Schedule 40 PVC pipe into 0.76 m (2.5 ft) lengths. A male threaded adapter was attached using PVC solvent cement on one end of each 0.76 m (2.5 ft) section and a female threaded adapter attached using PVC solvent cement onto the other end. This allowed for manipulation of discharge section length, elevation head, and equivalent pipe length.

The inlet section of each siphon consisted of a series of 90° elbows, a tee, a 50 mm (2 in) ball valve, and an adapter bushing in the 25 mm (1 in) and 38 mm (1.5 in) siphons (Figure 2). The 90° elbow allowed for the siphon barrel to rise to an elevation that represented the pond's desired full pool elevation. The ball valve allowed for manual startup and shutdown of each experiment. The adapter bushing reduced the pipe diameter going from the 50 mm (2 in) ball valve down to the 38 mm (1.5 in) and 25 mm (1 in) diameter siphons systems. Two inlet sections lengths indicated by “X” in Figure 2.3 were used and described in more detail later in this section.

Figure 2.2: Small Siphon Experiment Inlet Setup. Demonstrates how experimental siphons were constructed.

The barrel of each siphon system was fitted with a 3-way tee fitting to provide a siphon vent opening. The branch of the tee was fitted with bushings reducing it to a 13 cm (0.5 in) female NPT threads. For the flow tests, the vent was plugged allowing no air to enter the system. This allowed for flow testing to occur only during blackwater flow. Two 50 mm (2 in) 90° elbows were utilized to align the siphon outlet with the structural support used relative to the tank discharge position. See Figure 2.3 for more information on the setup of the experiments.

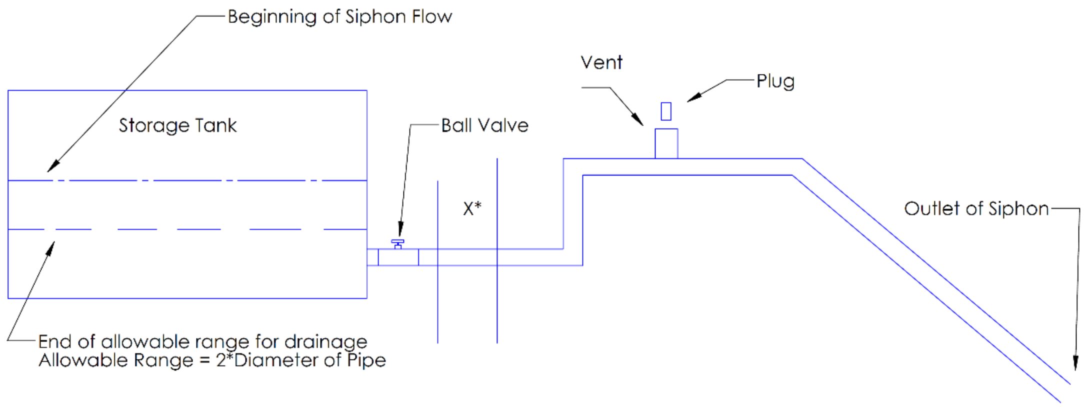

Figure 2.3: Experiment Overview Drawing. Shows specific set up of experiment.

Testing was conducted in two independent experiments. The first experiment was to quantify the discharge rate of a siphon as a function of hydraulic head and equivalent length of pipe.

<u>Siphon Discharge</u>

This was done by measuring discharge flow rate using the three small siphon systems previously described over the five stations (Figure 2.1) and two inlet section lengths (Figure 2.3). Hydraulic head was determined by surveying the elevation difference between the free water surface and discharge for each station. Equivalent length of pipe was as the sum of pipe length and equivalent lengths for fittings (PPI 2000). The fittings must be accounted for because they create resistance to water flow. Equivalent length for ball valve values was provided by Multiaqua (2006). Table 2.1 gives the values used to convert fittings into equivalent length of pipe.

Table 2.1: Fittings Friction Loss Conversion Table. Shows the values used to convert fittings to equivalent length of pipe for use in the models (Multiaqua 2006, PPI 2000)

	MM (IN)	MM (IN)	MM (IN)
DIAMETER OF PIPE	25.4 (1.0)	38.1 (1.5)	50.8 (2.0)
TEE	42.3 (1.6)	63.5 (2.5)	84.7 (3.3)
90° ELBOW	63.5 (2.5)	95.3 (3.8)	127 (5.0)
45° ELBOW	33.9 (1.3)	50.8 (2.0)	67.7 (2.7)
COUPLINGS	25.4 (1.0)	38.1 (1.5)	50.8 (2.0)
MALE/FEMALE INSERT ADAPTER	38.1 (1.5)	57.2 (2.3)	76.2 (3.0)
BALL VALVE	7.6 (0.3)	12.7 (0.5)	16.5 (0.6)

The invert elevation on the barrel of each siphon was constructed to coincide with the 1022 L (270 gal) elevation mark on the 1136 L (300 gal) tank, ensuring that after 114 L (30 gal) were discharged, the water level would be below the invert elevation of the barrel. The experiment was designed to collect during blackwater flow to quantify the average flow rates for each siphon system. The vent was plugged during observations to ensure siphons were in blackwater flow.

Discharge flowed directly into a 568 L (150 gal) discharge tank placed on a platform scale. The scale was composed of a retrofitted single axle trailer that was equipped with four Mettler Toledo 0745A load cells (Mettler Toledo Inc., Columbus, Ohio) positioned at the four corners of the trailer's square frame. The load cells were wired to a Phidgets Model 1046 USB bridge board (Phidgets Inc., Alberta, Canada) and the platform scale was calibrated up to a gross weight of 1040 kg (2300 lb). See Figure 2.4 for configuration of scale. Figure 2.5 demonstrates the scale calibration data.

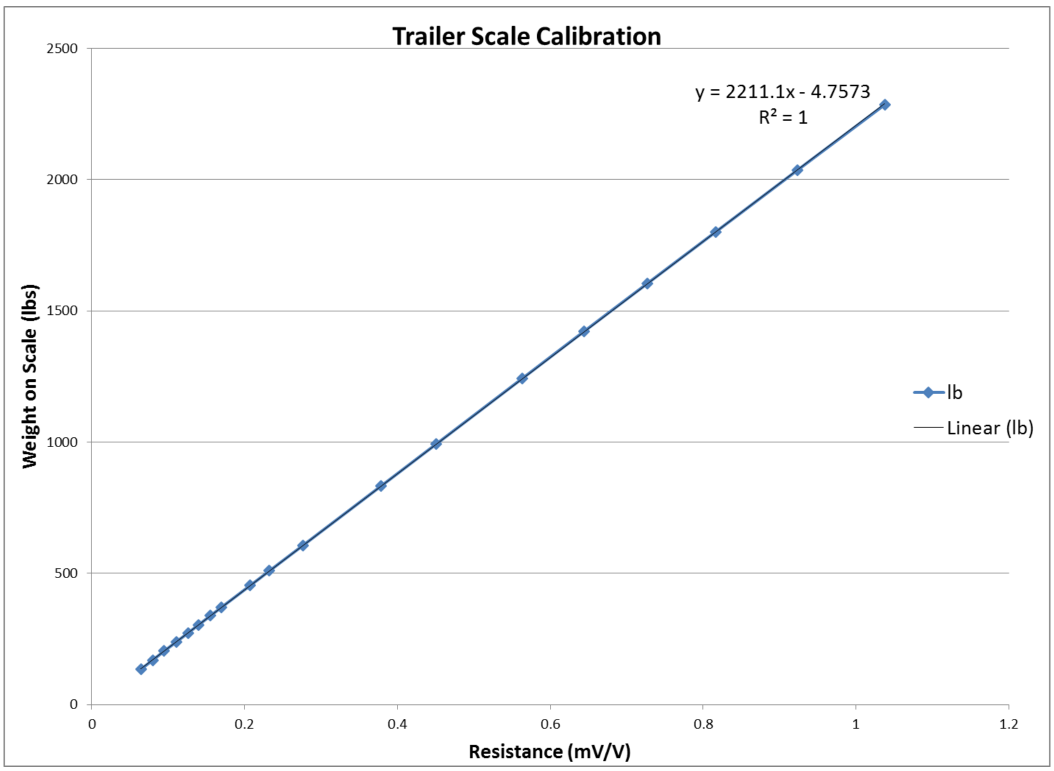

Figure 2.4. Trailer Scale Calibration Figure. Exhibits the weight increments and range across which the scale was calibrated. The calibration equation used for the data acquisition program is also included in the chart.

Figure 2.5: Trailer Scale Demonstration Pictures. Top picture shows scale trailer after development. Bottom picture shows the trailer during testing.

Siphon flow was directed to the discharge tank on the scale and a weight was recorded at a rate of 50 Hz during. This data was used along with water density to calculate average flow rate as change in over time during testing. All tests ran for 568 L (150 gal).

VENT SIZING

For required vent diameter tests, each vent plug was drilled to a prescribed diameter and flow was not measured. Observations recorded from the vent tests was the minimum vent diameter that prevented water from draining the tank below two pipe diameters of the designed normal water elevation.

Each vent trial was created by drilling a 13 mm (0.5 in) PVC plug. The vent sizes chosen provide about 0.2 mm (0.01 in) diameter increments in vent sizing. Criteria for minimum vent size required was defined as the minimum vent diameter which shut off discharge within two pipe diameters from the time of vent opening (Figure 2.3). Gauge marks were made on the tank to signify the desired water level for opening of vent and allowable range for continued siphon discharge. In the event that tank water level dropped below the allowable range, the vent size tested was determined to be too small and the next larger vent size test was evaluated until the minimum vent size criteria was met, which was recorded as the minimum required vent diameter.

For the vent tests, twenty vent sizes were used ranging from 1.0 mm (0.04 in) to 5.1 mm (0.20 in) as indicated in Table 2.2.

Table 2.2. Vent Sizes Used in Experiment. Shows vent diameters used in testing.

EXPERIMENTAL VENT SIZES

DRILL SIZE	Diameter mm (in)
7	5.1 (.20)
12	4.8 (.19)
15	4.6 (.18)
18	4.3 (.17)
21	4.0 (.16)
25	3.8 (.15)
28	3.6 (.14)
30	3.3 (.13)
31	3.0 (.12)
35	2.8 (.11)
39	2.5 (.10)
41	2.4 (.09)
43	2.3 (.09)
44	2.2 (.09)
46	2.1 (.08)
48	1.9 (.07)
50	1.8 (.07)
53	1.5 (.06)
56	1.2 (.05)
60	1.0 (.04)

RESULTS AND DISCUSSION

The results indicated that flow was proportional to elevation and pipe diameter, and inversely proportional to equivalent pipe length. The flow rate increases from increased elevation head from the increase of potential energy in the system. As pipe diameter increases, so does the area of which fluid can be transferred. Flow rate decreases from an increase in pipe length as a function of added resistance in the system. Results from Flow tests are provided in Table 2.3.

Table 2.3: Experiment Results from Small Siphons

Elevation Head Z m (ft)	**Length of Pipe L m (ft)**	**Inside Diameter of Pipe D mm (in)**	**Flow Q L/m (gpm)**	**Minimum Vent Diameter Required mm (in)**
4.88 (16.0)	46.68 (153.1)	52.50 (2.07)	300.4 (79.4)	4.3 (.17)
3.97 (13.0)	43.27 (142.0)	52.50 (2.07)	277.6 (73.3)	4.0 (.16)
3.43 (11.3)	39.85 (130.7)	52.50 (2.07)	249.6 (65.9)	4.3 (.17)
2.30 (7.5)	36.44 (119.6)	52.50 (2.07)	209.0 (55.2)	4.3 (.17)
1.30 (4.3)	33.03 (108.4)	52.50 (2.07)	148.4 (39.2)	3.8 (.15)
4.88 (16.0)	41.61 (136.5)	40.89 (1.61)	163.9 (43.3)	3.0 (.12)
3.97 (13.0)	38.61 (126.7)	40.89 (1.61)	145.4 (38.4)	3.0 (.12)
3.43 (11.3)	35.61 (116.8)	40.89 (1.61)	131.8 (34.8)	2.4 (.09)
2.30 (7.5)	32.62 (107.0)	40.89 (1.61)	110.1 (29.1)	2.3 (.09)
1.30 (4.3)	29.62 (91.2)	40.89 (1.61)	72.5 (19.1)	1.8 (.07)
4.88 (16.0)	34.17 (112.1)	26.64 (1.05)	59.2 (15.6)	2.2 (.09)
3.97 (13.0)	31.69 (104.0)	26.64 (1.05)	53.3 (14.1)	2.1 (.08)
3.43 (11.3)	29.20 (95.8)	26.64 (1.05)	52.6 (13.9)	2.1 (.08)
2.30 (7.5)	26.72 (87.7)	26.64 (1.05)	44.1 (11.7)	1.8 (.07)
1.30 (4.3)	24.24 (79.5)	26.64 (1.05)	36.5 (9.7)	1.5 (.06)
4.88 (16.0)	31.46 (103.2)	52.50 (1.05)	401.3 (106.0)	5.1 (.20)
3.97 (13.0)	28.04 (92.0)	52.50 (2.07)	372.5 (98.4)	5.1 (.20)
3.43 (11.3)	24.63 (80.8)	52.50 (2.07)	342.9 (90.6)	4.8 (.19)
2.30 (7.5)	21.22 (69.6)	52.50 (2.07)	324.7 (85.8)	4.8 (.19)

Elevation Head Z m (ft)	Length of Pipe L m (ft)	Inside Diameter of Pipe D mm (in)	Flow Q L/m (gpm)	Minimum Vent Diameter Required mm (in)
1.30 (4.3)	17.80 (58.4)	52.50 (2.07)	211.6 (55.9)	3.8 (.15)
4.88 (16.0)	25.92 (85.0)	40.89 (1.61)	210.4 (55.6)	4.3 (.17)
3.97 (13.0)	22.92 (75.2)	40.89 (1.61)	194.3 (51.3)	4.0 (.16)
3.43 (11.3)	19.93 (65.4)	40.89 (1.61)	178.1 (47.0)	4.0 (.16)
2.30 (7.5)	16.93 (55.5)	40.89 (1.61)	146.2 (38.6)	3.8 (.15)
1.30 (4.3)	13.93 (45.7)	40.89 (1.61)	137.9 (36.4)	2.8 (.11)
4.88 (16.0)	21.35 (70.0)	26.64 (1.05)	76.3 (20.2)	2.5 (.10)
3.97 (13.0)	18.86 (61.9)	26.64 (1.05)	68.4 (18.1)	2.4 (.09)
3.43 (11.3)	16.38 (53.7)	26.64 (1.05)	70.9 (18.8)	2.3 (.09)
2.30 (7.5)	13.90 (45.6)	26.64 (1.05)	62.5 (16.5)	2.1 (.08)
1.30 (4.3)	11.41 (37.4)	26.64 (1.05)	40.2 (10.7)	1.8 (.07)

Theoretically, all of the energy available in the elevation head should be accounted for as the sum of the velocity head and friction head. Velocity head was converted from gpm to m using the equations 2.1 – 2.5.

Equation 2.1 shows how the inside diameter of the pipe was used to calculate the cross sectional area of the pipe.

$$A = \frac{\pi D^2}{4} \qquad \text{(Eq. 2.1)}$$

Where A= cross-sectional area of pipe, in^2

D = inside diameter of pipe, in

Equation 2.2 shows how flow was converted from gpm to in^3/min.

$$Q_i = Q_g * 231$$ (Eq.

2.2)

Where Q_i = flow, in^3/min

Q_g = flow, gpm

Equation 2.3 shows how velocity was calculated using the cross sectional area of the pipe and flow rate.

$$v = \frac{Q_i}{A}$$ (Eq. 2.3)

Where V = velocity, in/ min

Equation 2.4 shows how velocity was converted from in/min to ft/s.

$$v_s = \frac{v}{.2}$$ (Eq. 2.4)

Where Vs = velocity, ft/s

Equation 2.5 shows how velocity head was calculated from velocity.

$$H_v = \frac{\left(\frac{Vs^2}{2}\right)}{32.1} \qquad \text{(Eq. 2.5)}$$

Where H_v= velocity head, ft

The data shows that as elevation head decreases, a lesser amount of it was accounted for in the velocity head and friction head. This could result from measurement errors or inaccurate assumptions for friction factors and equivalent length pipe lengths. This could also be an error from incorrect flow measurement. This analysis of theoretical and measured energy as related to siphon spillways has been a suggested topic of study (Gramatky, 1928) previously.

Table 2.4. Study of Energy Losses Table. Demonstrates energy conservation in experimental data. Demonstrates the head loss in as a function of friction head and velocity head over elevation head.

Study of Energy Losses				
Velocity head m (ft) H_v	Friction Head m (ft) H_l	Elevation Head m (ft) H_z	Sum of Measured Head (H_f+H_v) m (ft)	Measurement Error $1 - \frac{H_l + H_v}{H_z}$ %
0.27 (0.88)	4.42 (14.50)	4.88 (16.01)	4.69 (15.39)	-4%
0.23 (0.75)	3.54 (11.61)	3.97 (13.02)	3.77 (12.37)	-5%
0.19 (0.62)	2.68 (8.79)	3.43 (11.25)	2.87 (9.41)	-16%
0.13 (0.43)	1.76 (5.77)	2.30 (7.55)	1.89 (6.20)	-18%
0.07 (0.23)	0.85 (2.79)	1.30 (4.27)	0.91 (2.96)	-30%

Velocity Head m (ft) H_v	Friction Head m (ft) H_l	Friction Head m (ft) H_l	Sum of Measured Head (H_f+H_v) m (ft)	Measurement Error $1-\frac{H_l+H_v}{H_z}$ %
0.22 (0.72)	4.33 (14.21)	4.88 (16.01)	4.55 (14.93)	-7%
0.17 (0.56)	3.22 (10.56)	3.97 (13.02)	3.39 (11.12)	-15%
0.14 (0.46)	2.47 (8.10)	3.43 (11.25)	2.62 (8.60)	-24%
0.10 (0.33)	1.62 (5.31)	2.30 (7.55)	1.72 (5.64)	-25%
0.04 (0.13)	0.68 (2.23)	1.30 (4.27)	0.72 (2.36)	-44%
0.16 (0.52)	4.33 (14.21)	4.88 (16.01)	4.49 (14.73)	-8%
0.13 (0.43)	3.31 (10.86)	3.97 (13.02)	3.44 (11.29)	-13%
0.13 (0.43)	2.98 (9.78)	3.43 (11.25)	3.10 (10.17)	-9%
0.09 (0.30)	1.96 (6.43)	2.30 (7.55)	2.05 (6.73)	-11%
0.06 (0.20)	1.26 (4.13)	1.30 (4.27)	1.32 (4.33)	2%
0.49 (1.60)	5.09 (16.70)	4.88 (16.01)	5.58 (18.31)	14%
0.42 (1.38)	3.95 (12.96)	3.97 (13.02)	4.37 (14.34)	10%
0.36 (1.18)	2.98 (9.78)	3.43 (11.25)	3.34 (10.96)	-3%
0.32 (1.05)	2.32 (7.61)	2.30 (7.55)	2.64 (8.66)	15%
0.14 (0.46)	0.88 (2.89)	1.30 (4.27)	1.02 (3.35)	-22%
0.36 (1.18)	4.28 (14.04)	4.88 (16.01)	4.64 (15.22)	-5%
0.31 (1.02)	3.27 (10.73)	3.97 (13.02)	3.58 (11.75)	-10%
0.26 (0.85)	2.42 (7.94)	3.43 (11.25)	2.68 (8.79)	-22%
0.18 (0.59)	1.42 (4.66)	2.30 (7.55)	1.60 (5.25)	-30%
0.16 (0.52)	1.05 (3.44)	1.30 (4.27)	1.21 (3.97)	-7%
0.27 (0.88)	4.33 (14.21)	4.88 (16.01)	4.60 (15.09)	-6%
0.21 (0.69)	3.13 (10.27)	3.97 (13.02)	3.34 (10.96)	-16%
0.23 (0.75)	2.90 (9.51)	3.43 (11.25)	3.13 (10.26)	-9%
0.18 (0.59)	1.95 (6.40)	2.30 (7.55)	2.12 (6.96)	-8%
0.07 (0.23)	0.71 (2.33)	1.30 (4.27)	0.78 (2.55)	-40%

Multiple linear regressions for predicting flow rate and minimum vent diameter required were developed in JMP v1.7593.38 (Sun Valley, California) as functions of 1. Pipe diameters, 2. Head differentials, and 3. Equivalent pipe lengths. An issue with early statistical analyses showed a heteroscedasticity, or tail flare, in the results when plotting Q residual vs Q predicted for the physical-characteristics-based model with untransformed regressors Z, D, and L. The resulting plot showed prediction error increasing with decreasing elevation head. Measurement errors potentially resulted from entrance/exit losses that were not accounted for in this experiment. The error would be magnified when extrapolating the data for use in predicting flow on a large pipe that might be used in regulating water level in a pond or waterbody.

In an attempt to reduce heteroscedasticity, Bernoulli's Principle was used as a physical model on which to base the model. Bernoulli's principle explains flow of fluids through conservation of energy. Equation 2.7 shows the relationship between elevation head, head loss and velocity head.

$$H_z - H_l = H_v \qquad \text{(Eq 2.7)}$$

Where H_z = elevation head (dimensionless)

H_l = head loss (dimensionless)

H_v = velocity head (dimensionless)

For the purposes of this model the following proportionalities were assumed as shown in Equations 2.8 – 2.10. These equations demonstrate between elevation head and change in elevation, head loss and equivalent length of pipe, and velocity head and velocity.

$$H_z = \ Z \qquad \text{(Eq 2.8)}$$

Where Z = elevation, m

H_z = elevation head, in

$$H_l = \ LV^2 \qquad \text{(Eq. 2.9)}$$

Where H_l = head loss in pipe, m

L = equivalent length of pipe, ft

V = velocity, m/s

$$Hv = \ V^2 \qquad \text{(Eq. 2.10)}$$

Substituting Equations 2.8, 2.9, and 2.10 into Equation 2.7 provides Equation 2.11.

$$Z - LV^2 \propto V^2 \qquad \text{(Eq. 2.11)}$$

Rearranging and reducing Equation 2.11 provides the following proportionality shown in Equation 2.16.

$$\frac{Z}{L} \propto V^2 \qquad \text{(Eq. 2.12)}$$

Since pipe velocity is equal to flow rate divided by cross sectional area and cross sectional area varies with the square of pipe diameter, Equation 2.13 can be rearranged.

$$Q/D^2 \propto V \qquad \text{(Eq. 2.13)}$$

Where Q = flow rate, m^3/s

=

Through substituting Q/D^2 for V in Equation 2.13 into Equation 2.12 and solving for flow rate, Equation 2.14 is established, providing a relationship between variables measured for this model development and flow rate.

$$Q \propto D^2 \sqrt{\frac{Z}{L}} \qquad \text{(Eq. 2.14)}$$

For modeling purposes, the "β" term was defined from Equation 2.14 to be used as a model regressor as shown in Equation 2.15.

$$\beta = D^2 \sqrt{\frac{Z}{L}} \qquad \text{(Eq. 2.15)}$$

Application of β in subsequent linear regression analysis resulted in reduction of heteroscedasticity as compared to that previously discussed.

Equation 2.16 based upon Bernoulli's Equation gave the greatest coefficient of determination and least root mean square error when predicting flow from the data collected and is as follows:

$$Q_g = -18.05 + 18.82 * \beta + (\beta - 3.02) * ((D - 1.58) * 7.88) \qquad \text{(Eq. 2.16)}$$

Figure 2.6 shows the residual of a flow prediction equation from Equation 2.16 with a 95% confidence interval with measured flow (Q actual) on the y axis and predicted flow from the β model on the x axis. Using a physical model to build the equation held the data close to predicted values within the range of pipe diameters that were used in the experiment as demonstrated on the previous graph.

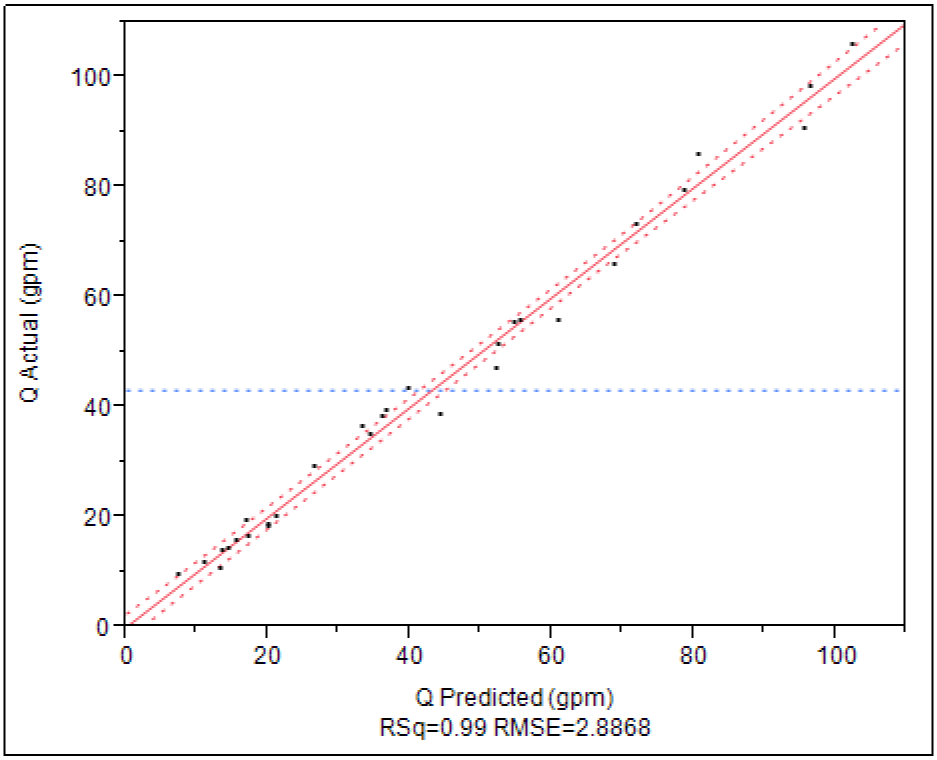

Figure 2.6: Actual vs. Predicted Flow Rate for β Model. Graph of predicted flow from the Bernoulli's based model vs recorded flow over the small siphon spillway data points collected during experimentation.

The other flow rate model developed was based on physical characteristics measured during the data collected from te small siphon experiments, including pipe diameter, length of pipe and elevation head. Equation 2.17 represents the model based on physical characteristics

$$Flow= Z(0.2483) + L(-0.2784)+D(66.9)-68.12 \quad \text{(Eq. 2.17)}$$

Where Z = elevation head, in

L = equivalent length of pipe, ft

D = inside pipe diameter, in

The results of measured flow (Q actual) vs. predicted flow (Q predicted) from equation 2.17 are displayed in Figure 2.7.

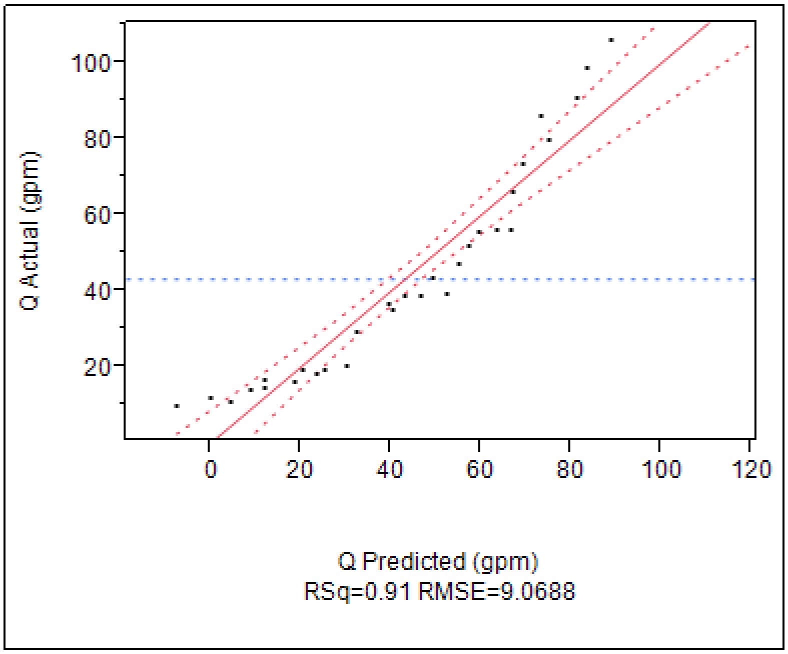

Figure 2.7. JMP output for hydraulic characteristics based model, graph of predicted flow based on hydraulic characteristics vs recorded flow over the small siphon spillway data points collected during experimentation.

Comparison of Figures 2.6 and 2.7 show that in both models as flow rate increases the accuracy of the model is reduced. Figure 2.7 shows the equation based on hydraulic characteristics has data points from the experiments increasingly outside of the confidence interval as flow rate increases. Comparison of these figures show Figure 2.6 predictions were closer to the measured flow rate than when compared to Figure 2.7.

VENT RESULTS

Regression models were also developed for the vent test data. During the analysis, it was determined that the most accurate statistical model was by predicting minimum vent size as a function of the natural log of flow. This model estimated the minimum required vent opening for siphon spillways from the data collected during the small siphon experiments. Equation 2.18 is the model for predicting the minimum required vent opening on a siphon.

$$d_v = -.0815+.0590(\ln(Q)) \qquad \text{(Eq. 2.18)}$$

Where d_v= minimum vent diameter, in

Once the data had been analyzed, the flow model and vent model were tested for validation using a siphon that was installed on LaMaster Dairy Farm. Specifications of the siphon were measured to input as variables used in the model derived from the data collected from the experimental siphon tests. Pipe length was measured to be 28.4 m (99.3 ft) of 200 mm (8 in) Schedule 40 PVC, along with the fittings recorded in Table 2.5 and Table 2.6. Inlet and outlet losses were unmeasured in this assessment of siphon spillways

Table 2.5. LaMaster Siphon Equivalent Pipe Length of Fittings. Shows the conversion from fitting to equivalent length of pipe. L_f value is the equivalent length of pipe conversion total for each fitting type.

Fitting	Amount	Equivalent Pipe Length/Fitting mm, (in)	Lf Value mm, (in)
45°Elbows	6	269.2 (10.6)	1615.4 (63.6)
Tee (main)	1	337.8 (13.3)	337.8 (13.3)
Couplings	4	203.2 (8.0)	812.8 (32)
	Total Equivalent Length for all fittings used mm, (in)		2766.0 (108.9)

Table 2.6. Total Equivalent Pipe Length for LaMaster Siphon. Shows length of pipe including fittings using equivalent pipe length method. Sum is equal to L variable for LaMaster data point.

Equivalent Pipe Length Totals for LaMaster Siphon m, (ft)	
Equivalent length for fittings	2.8 (9.1)
Length of straight pipe in siphon	30.27(99.3)
Total equivalent pipe length	33.0 (108.4)

The flow test at LaMaster was initiated by manual priming of the siphon spillway. A Model PT-I-40-8.0 saddle mounted flow meter with reported accuracy of +/- 1.5% was installed for measuring discharge flow on the siphon spillway (Midwest Instruments and Controls, Rice Lake, WI). Elevation head at the time of the test was 6.28 m (20.6 ft) and flow was measured to be within a range of 6275-6900 L/min (1650 – 1825 gpm) with an average of 6585L/min (1740 gpm). Table 2.7 shows that the measured flow from the siphon at LaMaster was not near the estimates formulated from the small siphons.

Table 2.7. Flow Prediction Model Testing. Shows the predicted values for flow with the physical parameters that were measured from the LaMaster siphon system. Measured flow was the average flow rate recorded during testing on the LaMaster siphon system.

Flow Prediction Model Testing		
Physical Model estimate	B Model estimate	Measured Flow
Estimate L/min , (gpm)	Estimate L/min (gpm)	L/min (gpm)
1881 (496)	24583 (6496)	6585 (1740)

Several factors could have influenced the model's failure to estimate flow accurately on the LaMaster siphon spillway model. The primary factor is likely that the small siphons created an equation that was extrapolated to estimate flow for more than one order of magnitude greater than that in small siphon testing. Another possible factor is that entrance and exit losses were not evaluated in the experimental siphon or LaMaster siphon.

Vent tests on the LaMaster siphon included 5 vent sizes 50.8 mm (2 in), 25.4 mm (1 in), 19.1 mm (0.75 in), 12.7 mm (0.50 in), and 6.4 mm (0.25 in). Each test was initiated by manual priming of the siphon. As vent diameter decreased in size, the amount of time needed for shut off of siphon increases. The vent tests at LaMaster showed that a 12.7 mm (0.5 in) diameter vent was sufficient for shut off of a 200 mm (8 in) siphon. The tests also indicated that a 6.4 mm (0.25 in) opening was not enough to break the siphons cycle. The siphon ran for 30 hours and drained 0.79 m (2.6 ft) without interruption with the 6.4 mm (0.25 in) vent opening. The siphon had to be manually shutoff to prevent the pond from draining further. Table 2.8 details the results from the vent tests.

Table 2.8. LaMaster Vent Tests Results. Pipe diameters drained were calculated by multiplying the amount of time the siphon discharged by the average flow rate measured in the flow rate experiment on LaMaster Dairy. Results were also validated by survey after siphon disengaged.

Shutoff Time of Siphon

Vent Size mm (in)	Shutoff Time	Pipe Diameters Drained
50.8 (2.0)	< 10 s	0.00
25.4 (1.0)	10 s	0.00
19.1 (0.75)	30 s	0.00
12.7(0.50)	4 min	0.01
6.4 (0.25)	Did not shutoff	>3.9

The equation for predicting minimum vent size required estimated that a 9.1 mm (0.36 in) vent opening would be required to stop the siphon from continuing to discharge. The minimum required vent opening model was validated when the 6.4 mm (0.25 in) vent opening could not shut off the siphon within parameters of the test and the 12.7 mm (0.50 in) vent opening shut off discharge within the test parameters. Figure 2.8 shows the results with the prediction formula represented on the graph as well.

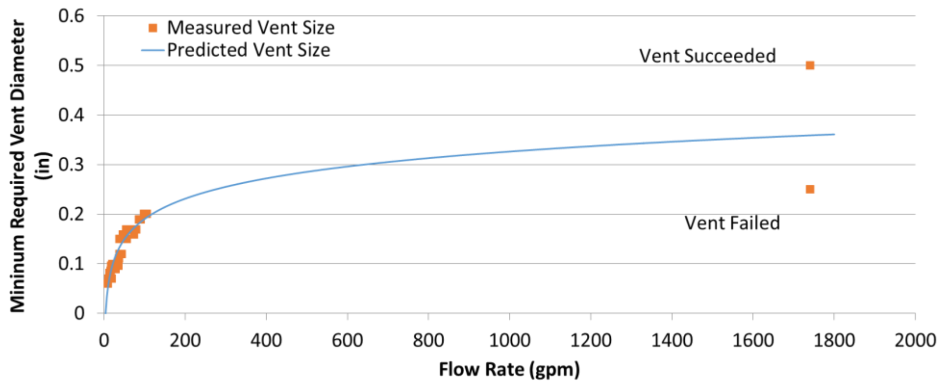

Figure 2.8. Vent Model Validation Graph. Graph showing results of LaMaster siphon and small siphons. Graph represents data collected during small siphon testing and LaMaster siphon testing. Equation 2.18 was used to predict required vent size.

The vent tests on the siphon at LaMaster did shed some light on the ponds that were drained by a siphon spillway. With a 12.7 mm (0.5in) vent opening sufficing to break the siphon cycle, it practically rules out the possibility that the ponds were drained because a 50.8 mm (2 in) vent was inadequate, which is the recommended vent opening from NRCS for siphons up to 203 mm (8 in) . An alternative cause of the overdrainage could be clogging of the vent opening by trash or aquatic weeds during a discharge event, reducing the amount of air that can enter the system.

CONCLUSIONS

Empirical models were created for prediction of flow rate and minimum required vent diameter based on data collected during small siphon spillway tests. The emperical model for flow was shown to be inaccurate after extrapolating to a large siphon system.

For refinement of the flow model, the physical parameters and flow rate of the siphon spillway at LaMaster was included in the flow prediction model based on β using the same model structure, Equation 13 was generated and is as follows:

$$Q = -13.75 + 18.84\,\beta + (d-1.78) * (\beta-6.03)(-0.1) \qquad \text{(Eq. 2.19)}$$

Figure 2.9 shows the refined flow prediction equation with the data from the small siphons and the data point from LaMaster included on the plot with measured flow (Q actual) on the y axis and predicted flow (Q predicted) on the x axis.

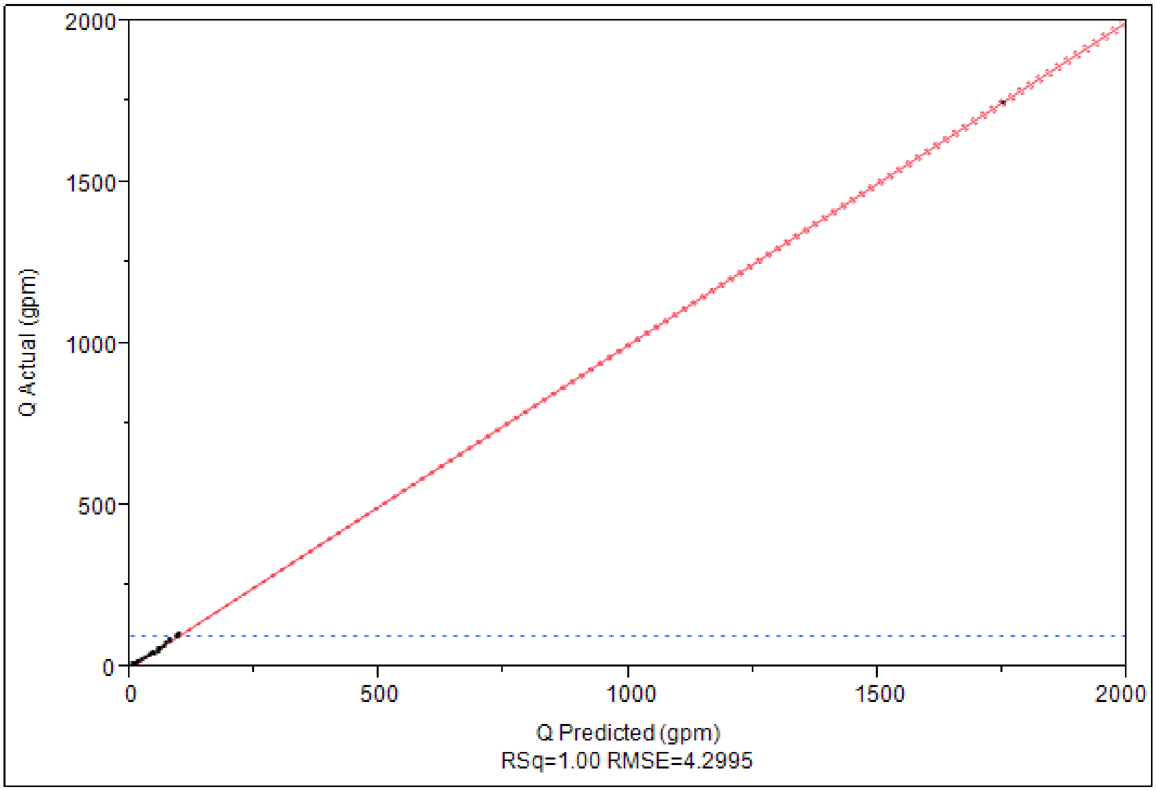

Figure 2.9. JMP output for refined flow equation based on β. Graph shows the fit of the refined flow prediction equation.

Further testing on large siphon spillways should be done to improve the flow prediction models. Each data point on a real world siphon would help add robustness to the data set and provide a more precise flow prediction equation as well as provide further validation of the model structure. A precise flow prediction equation based on hydraulic characteristics for siphon spillway would be an immense use to landowners and property managers.

The empirical model built to predict minimum vent opening diameter was tested and validated at the LaMaster siphon. More testing could be done to further validate this model. This test demonstrated that a 51 mm (2 in) vent opening is more than sufficient to shut off a 200 mm (8 in) siphon at 6 m (20 ft) of elevation head. A vent guard would help protect a siphon spillway from malfunctioning. Below is an example of a vent guard should be considered for testing composes a 50.8 mm (2 in) PVC vent with a length of 100 mm (4 in) PVC covering the vent. The outer cover should have openings drilled to equal four times the cross sectional area of the vent opening to protect against clogging of the vent. Figure 2.10 illustrates the proposed vent guard

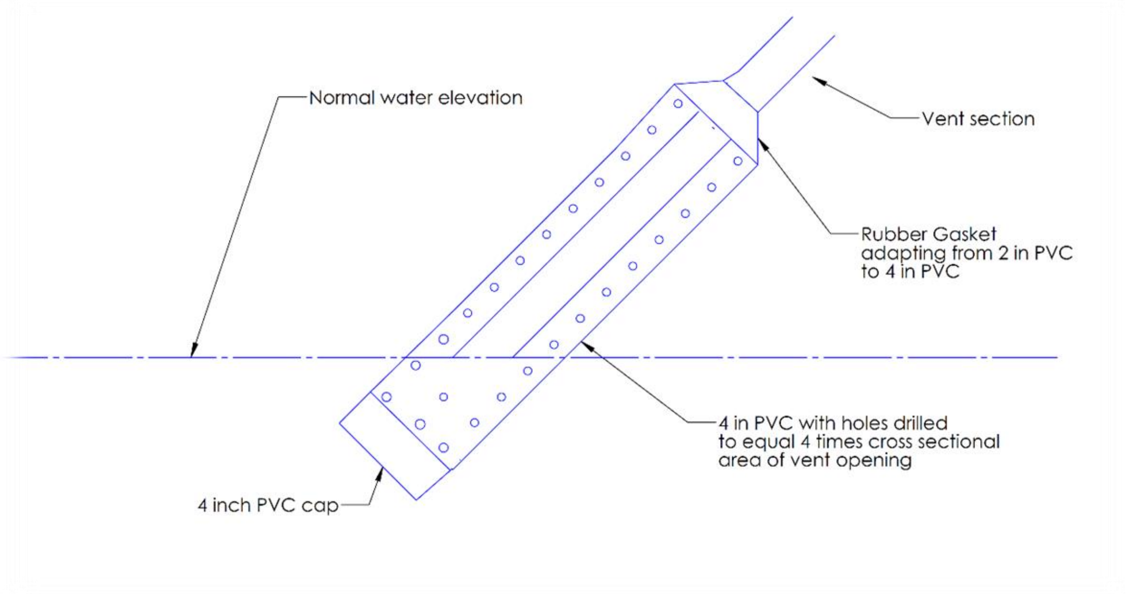

Figure 2.10. Vent Guard Figure. Shows the concept of a vent guard. Should be considered for testing in future field studies.

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CHAPTER 3

GUIDELINES FOR DESIGN AND CONSTRUCTION OF SIPHON SPILLWAYS

ABSTRACT

A common problem in pond performance is deterioration and reduced functionality of the primary spillway. Many water control structures are nearing the end of their materials' life expectancy. Primary spillways constructed of corrugated metal pipe have a life expectancy of thirty years (Montana DNR, 2012). The objective of this project is 1. To develop a set of design and construction guidelines for pond spillway rehabilitation using an air-regulated siphon spillway system. This type of water control structure will be useful to landowners as an economical and reliable replacement for vertical riser spillways.

To examine the application of the guidelines outlined in this chapter, a failed 40 year old water control structure at LaMaster Dairy Farm on the campus of Clemson University was replaced with a siphon spillway. The existing riser pipe rusted off approximately 1 m (3 ft) below its normal design elevation, resulting in an estimated loss of 1.5 ha m (16 ac ft) of storage. Using a 2 year, 24 hour peak runoff rate from the 47.8 ha (118 ac) pond drainage area, calculations showed an 8-inch siphon would be conservatively sufficient. As part of the guidelines developed here, Natural Resources Conservation Service (NRCS) specifications and standards are used in curve number estimation, siphon sizing and elevation, runoff calculations and emergency spillway design. Construction procedures are also reviewed in this paper. The guidelines are a tool to show the correct steps in designing and constructing a siphon spillway.

INTRODUCTION

Many ponds were built throughout the United States in the early 20th century. Pond uses include, but are not limited to, irrigation for crops, water source for livestock, and recreation. Common materials used for water control structures, such as corrugated metal pipe, have a lifespan of 25-30 years (Montana DNR, 2012). The earthen dam will still be in place as the corrugated metal that controls the water level oxidizes and degrades. Eventually such a failure lowers the elevation of the primary spillway which controls the water level of the pond. Many water control structures function well past their life expectancy. Seasonal weather patterns may lead to fluctuations in water level that could weaken many primary spillways. The chance for degredation of a water control structure increases when the material is cycled between wet and dry stages (Veesaert, 2006). When the water level is consistent, pipe deterioration is reduced by the lack of atmospheric oxygen. During periods of drought, the elevation of the water level in many ponds decrease, leaving the corrugated metal spillway open to the atmospheric oxygen. Degradation of water control structures have created an issue with control and maintenance of these waterbodies.

The price of construction is often related to the price of fuel (A. Hajji, 2013). Alternative methods that reduce the amount of earthwork have to be investigated with respect to constraints on project funding. The method of replacing a deteriorated spillway with the same type of vertical riser spillway requires a section of the dam to be completely removed and rebuilt. The water body must be drained for the project to be completed. New spillway pipe is put in below bottom of dam and an earthen dam must be rebuilt around new pipe.

An alternative spillway design such as air-regulated siphon spillway reduces the amount of earthwork and labor required since it is buried at the elevation of normal pool for the waterbody. With the installation of a vertical riser, the waterbody will be completely drained. This makes siphon spillways a more viable option if maintaining aquatic life and pond functionality during the project is a goal.

Siphons have been used for hydrologic control for over 100 years (Ackers, 2000). The siphon system works by having water pull itself out through the pipe. After water starts to rise above the normal pool elevation and closes the circuit by covering the vent hole, the water begins to pull itself out of the waterbody and through the pipe. If designed and maintained properly, the siphon spillway is self-priming and efficient (Alabama NRCS, 2009).

There are four stages of flow in a siphon spillway. The flows increase as stages progress from weir flow to subatmospheric weir flow to partial flow to blackwater flow (Babaeyan-Koopaei et al., 2002). As flow increases air in the system is decreasing, allowing for additional water to be discharged through the barrel. The vent on a siphon spillway is important for breaking the siphonic cycle and depriming the siphon. The vent allows air to enter the system when the water level has dropped below full pool elevation. Without a vent opening, siphon spillways would continue to be engaged until the waterbody is lowered to the invert of the inlet. Georgia NRCS (2012) siphon drawings shows the vent elevation and the invert of the barrel to be the same.

Choosing a sufficient material for the replacement primary spillway that has a long life expectancy is an important aspect of pond rehabilitation. While corrugated metal only has a lifespan of 25-30 years, other materials have a much longer service life. Concrete pipe has a lifespan of 100 years or more. The life expectancy when PVC is properly maintained is estimated to be 100 years. PVC has been used for hydraulic functions since 1958 (Vibean, 2009).

OBJECTIVES

1. Develop design procedures for a siphon spillway
2. Develop construction procedures for a siphon spillway
3. Demonstrate design and construction procedures through LaMaster siphon example
4. Evaluate cost effectiveness through budget analysis example

Procedures for Design of Siphon Spillway

The first step for rehabilitation of water control structures is to determine the volume of water that needs to pass through the structure. In the absence of original plans or drawings of the pond from the time of construction, an analysis of hydrology is needed. Watershed area can be determined using a United States Geological Survey quadrangle topography map or mapping software with a topography layer of the area. After the watershed area is determined, the watershed length and watershed slope must be measured or calculated. The watershed length is the longest distance within the watershed, with respect to flow and travel time, that a drop of rain would have to travel before reaching the dam in a rainfall event. The watershed slope is average change in elevation over distance through the watershed. More information can be found on how to delineate a watershed at New Hampshire NRCS (2003).

The weighted runoff curve number (CN) can be obtained by delineating land uses within the defined watershed. Curve numbers come from the NRCS TR-55 methodology, Iowa NRCS (2008). The curve number is a function of land use within the defined watershed, amount of infiltration, soil type, and water table. The higher the curve number is, the more runoff that is produced during each storm event. Land use affects infiltration, which alters the curve number. A pasture with good grass would have a much lower CN than a parking lot with the same parameters. Soil type plays an important role, as sandy soils allow more infiltration and clay dominant soils produce more runoff. There are four hydrologic soil groups in the curve number function (Group A, Group B, Group C, and Group D). The groups are separated from lowest to highest amount of runoff. Group A soils are well drained sandy soils with potential for high infiltration rates so the runoff curve number would be lower than that for Group D soils, which are clay soils with high water tables and potential for high runoff. In Table 3.1 below, a sample weighted curve number calculation is provided, where the sum of the A*CN column, divided by the sum of the area column is equal to the weighted curve number. Curve number tables are available from NRCS (Iowa NRCS, 2008).

Table 3.1. Sample weighted curve number calculation table.

Land Use	Soil Hydrologic Group	Area (ac)	CN	A * CN
Row crop, Good	Group B	5	81	405
Pasture, Poor	Group B	2	79	158
Pasture, Good	Group A	10	39	390
Sum		17		861
Weighted CN	56.1			

After obtaining the weighted CN and watershed size, peak runoff rate was calculated using the TR-55 method found in NRCS Handbook Section 4, Hydrology (2008). The TR-55 method is functional for watershed areas that are less than 810 ha (2000 ac). This method includes a series of equations that estimate peak runoff rate for a given storm return period (1 year, 5 year, 10 year, 25 year, 50 year, and 100 year). A decision must be made about what return period storm event the emergency spillway should be designed to safely pass. This decision is a function of budget, cost of failure, and hydrology. The size of the storm event that the spillway is designed to handle dictates the size of the spillway to be installed. Designing for a larger storm event reduces the frequency of use of the emergency overflow spillway and therefore reduces associated maintenance on the emergency overflow spillway.

The equations below are steps to predict the runoff and unit peak discharge from a given rainstorm event. Equation 3.1 gives the user time of concentration from the rainstorm event. Equation 3.2 determines the soil-water retention parameter to be used in a later equation. Equation 3.3 estimates the amount of runoff volume in inches. Equation 3.4 estimates the runoff from a rainstorm event in cfs/in/mi^2. Equation 3.5 converts the output of Equation 3.4 from cfs/in/mi^2 to cfs. Equations 3.1- 3.5 were found in Schwab, et al. (1996).

$$tc = \frac{L^{0.8}\left(\frac{1000}{CN} - 9\right)^{0.7}}{1140 \cdot s^{0.5}} \qquad \text{(Eq. 3.1)}$$

Where tc = time of concentration (hr)

L = length of slope (ft)

s= average slope of the watershed (%)

$$S = \frac{1000}{CN} - 10 \qquad \text{(Eq. 3.2)}$$

Where S = maximum soil water retention parameter (unitless)

CN = Curve number calculated earlier in hydrology study (unitless)

$$Q = \frac{(P - 0.2 \cdot S)^2}{P + 0.8 \cdot S} \qquad \text{(Eq. 3.3)}$$

Where Q = runoff volume (in)

P = precipitation (in)

$$\text{Log}(q_u)\ 2.51 - 0.7(\log(tc)) - 0.15(\log(tc))^2 + 0.071(\log(tc))^3 \qquad \text{(Eq. 3.4)}$$

Where q_u = unit peak discharge (cfs/in/mi^2)

$$q = (q_u)(Q)(A) \qquad \text{(Eq. 3.5)}$$

Where q =peak runoff rate (cfs)

qu = unit peak discharge (cfs/in/sqmile) from Equation 3.4

Q = runoff volume (inches), from Equation 3.3

A = area in watershed (mi^2)

The emergency spillway width is found by using Equation 3.6 (Schwab et al., 1996). Local and state regulations often dictate the return period storm to be used in designing a pond.. A higher hazard class dam may require a longer return period used for design.

$$W = q / 3.2 \qquad \text{(Eq. 3.6)}$$

Where W = bottom width of the spillway, ft

q= peak runoff rate, from Eq. 3.5, (cfs)

The invert elevation of the siphon spillway barrel defines the normal pool elevation. The barrel of the siphon spillway must be placed where it will receive pressurization from the water level of the pond being above it during flood storage conditions. This pressurization comes from the head of the water in flood storage that is stacked prior to being discharged through the emergency spillway. Normal placement of the barrel is at the proposed normal pool elevation (Georgia NRCS, 2012). This ensures that flow will come through the water control structure during storm events.

Designing the siphon spillway is a function of flood storage and size of the pond. Flood storage volume of a pond is the volume stored between normal pool elevation and emergency spillway elevation. This elevation difference can be multiplied by the acreage of the pond provides the volume of flood storage for the pond. A siphon for a pond should be sized to discharge the flood storage above normal pool of the pond within ten days. (Alabama NRCS, 2009). Therefore, the required flow rate is equal to the flood storage volume divided by ten days.

The peak flow rate calculated using Equation 3.5 can be used with Figures 3.1, 3.2, and 3.3 to assist in selecting pipe size for use in siphon spillways. Converting fittings and straight pipe to equivalent pipe length is required. These charts should be used as an estimate as the equation has not been fully validated.

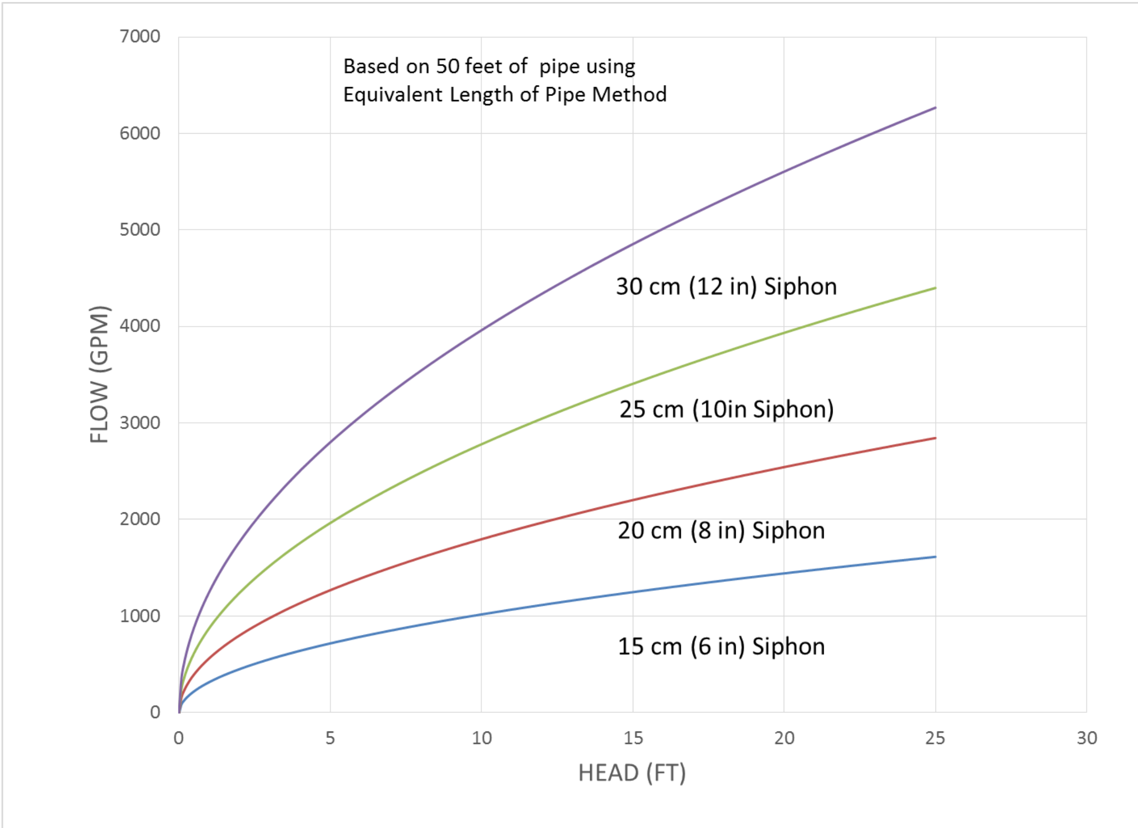

Figure 3.1. Flow Prediction and Pipe Sizing based on 15 m (50 ft) of pipe. Chart developed to size siphon spillways by amount of head and the desired discharge of the system. This chart used 15 m (50 ft) of pipe for the L variable in Equation 2.19.

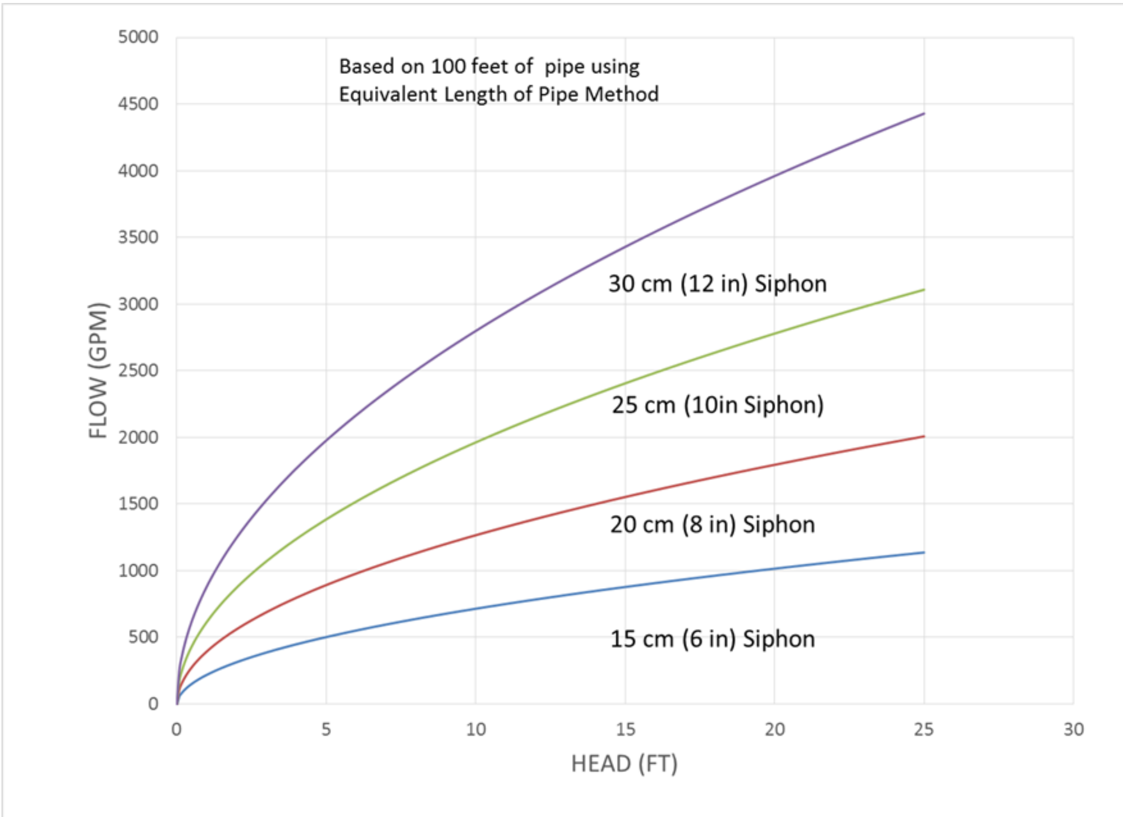

Figure 3.2. Flow Prediction and Pipe Sizing Chart based on 30 m (100ft) of pipe. Chart developed to size siphon spillways by amount of head and the desired discharge of the system. This chart used 30 m (100 ft) of pipe for the L variable in Equation 2.19.

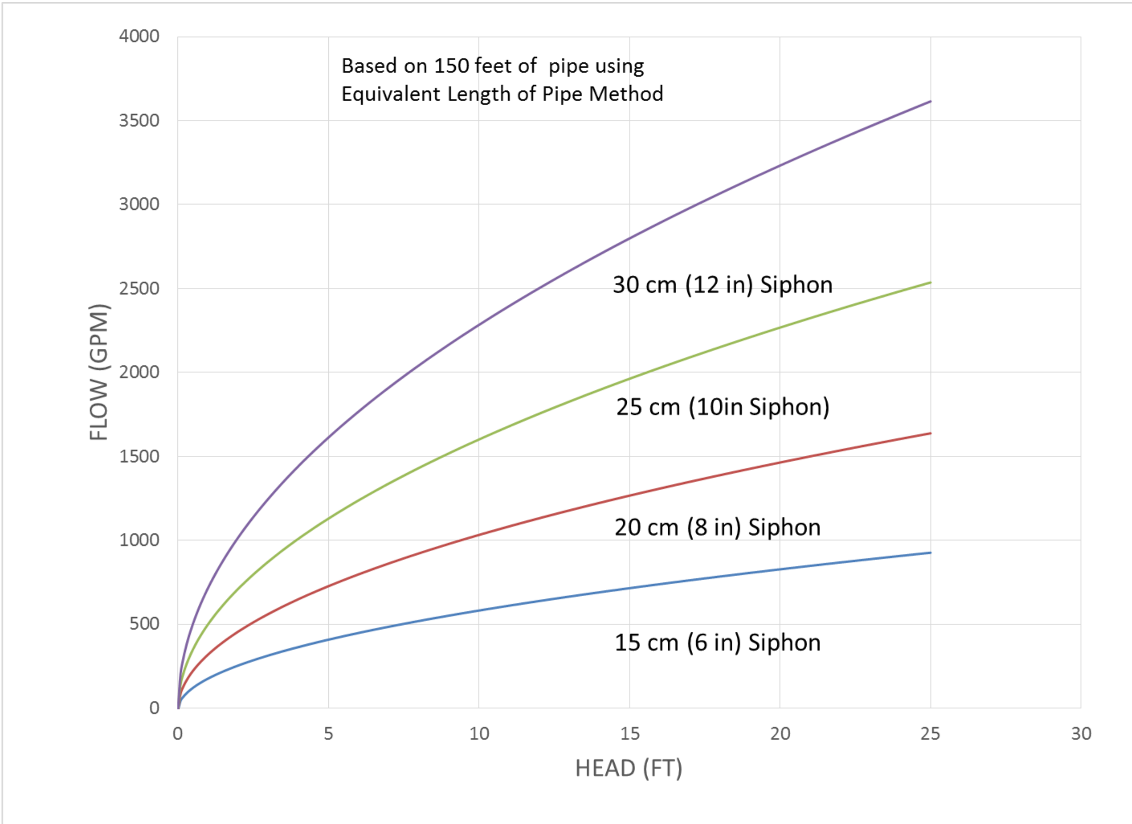

Figure 3.3. Flow Prediction and Pipe Sizing Chart based on 46 m (150 ft) of pipe. Chart developed to size siphon spillways by amount of head and the desired discharge of the system. This chart used 46 m (150 ft) of pipe for the L variable in Equation 2.19.

<u>Procedures for Construction of Siphon Spillway</u>

After the design of the siphon spillway, installation of the spillway begins. This can be done one of many ways. For pond rehabilitation projects, the first step is to lower the pond depth. Many ponds have gate valves that can be opened to drain the pond. A pump or temporary siphon can also be used to discharge water from the pond. A pump can discharge excess water quickly, but must be monitored. Fuel costs for a pump could be prohibitive. Using a small siphon, once primed, will discharge a flow of water continuously until water is below its inlet. Increasing pipe diameter used in the temporary siphon will increase flow rate through the system. A temporary siphon requires labor and flow valves in the inlet and outlet sections for proper charging as the pipe will be placed above top of dam. Having a ball valve on both ends of the temporary siphon allows for the pipe to be manually filled with water for charging and startup. A 50 mm (2 in) temporary PVC siphon has a lower discharge rate than most engine-driven pumps, but could be much more economical if a pump is not available for the project.

Once the pond is drained to a level below where the siphon is to be installed, earth moving can begin. A backhoe or mini-excavator can be utilized to remove a cross-section channel through the dam. It is important to have an experienced equipment operator for the installation of the siphon spillway. An experienced equipment operator can limit the area impacted during earthwork. Also, an experienced operator can limit the amount of soil that will be needed to repack the cross section by removing the soil in a manner that allows most of it to be used in the repacking stage. Surveying equipment can be used in conjunction with the backhoe to construct earthwork to the exact elevation required. This requires a backshot at the top of dam and several foresites along the cross-section through the dam during earthmoving to get the correct level for the siphon barrel conduit. Once earthwork has been completed satisfactorily, the next step is to begin laying the pipe.

Laying the pipe is a manual process, but should be assisted with equipment for 200 mm (8 in) and larger PVC pipe as was encountered at the demonstration site discussed later. The pipe can come in 6.1 m (20 ft) sections or 3.0 m (10 ft) sections. Once the horizontal barrel of the siphon is placed within the dam, both upstream and downstream sections can be installed. The siphon spillway can be placed above or below the side slopes of the dam using trenching or braces. When placed above side slopes, the siphon must be braced to protect from pipe flexing. For surface placement of the siphon spillway, UV protection will also be required.

On the intake side of the siphon spillway, a reducer tee joint will have an opening for a 50 mm (2 in) or 100 mm (4 in) vent as needed by design. The vent is a smaller pipe that leads from above the main barrel of the siphon and follows the layout of the barrel and inlet sections to the designed water level elevation. The vent establishes the water level for the new primary spillway. Once water goes above the vent, the siphon begins to produce suction in the barrel pipe from the water flowing through the inlet, which begins to discharge water through the pipe from the waterbody. Flow will continue until air enters the system. The end of the vent should be cut along the horizon at the specific height elevation where the designed full pool elevation. Surveying equipment should be used to obtain the precise placement.

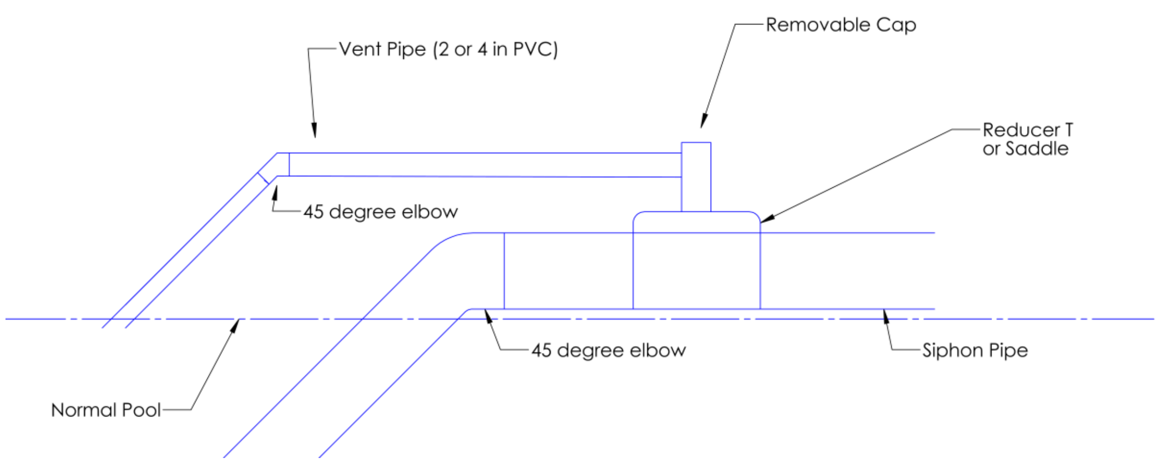

Figure 3.4. Air Regulated Siphon Vent Figure Configuration from Georgia NRCS, 2012

Both ends of the spillway should have some type of screen or guard. Without some type of guard, the inlet could become clogged with mud or vegetation and not be able to engage properly. The method of screen used on the inlet on this project was to put an end cap on the inlet and drill holes throughout a section of the inlet (Figure 3.2)**.** This allows water to take the path of least resistance during the discharge process and minimize the risk of clogging. The area of holes in the inlet should be equal to four times the cross sectional area of the pipe opening that was capped (Georgia NRCS, 2012). Keeping the entire inlet intake below the water level is important as well. If air enters the siphon system, the system may stop discharging water. The guard described also keeps wildlife such as turtles from lodging within the pipe and causing resistance to flow. It is also imperative to create a guard on the outlet end of the spillway. This will keep the larger animals from going inside the pipe. These minor modifications can keep obstructions from becoming severe problems and decreasing functionality of the spillway.

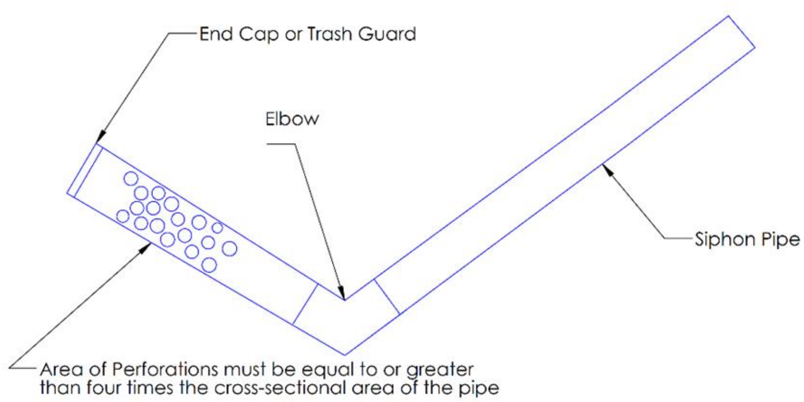

Figure 3.5. Inlet Guard figure adapted from Georgia NRCS, 2012

Once the siphon spillway is in place, it is time to cover and brace the pipe. A backhoe is suggested to cover and compact the soil around the pipe. PVC pipes obtain their strength from the fill surrounding them (Ohio DNR, 2009), thus having a compacted, stable surrounding is imperative. After the area around the pipe has been compacted, there should be an extra five to ten percent allowance of fill for settling at the top of dam. This keeps the area from becoming a low point in the dam, creating a wet or weak spot in the dam. There may be a need for more fill dirt to be brought in for maintenance of the area periodically depending on traffic on the dam.

If not buried, after fill, there is a need to brace the siphon spillway on the slopes of the dam. Braces can be made to either hold the pipe to the ground or to hold the pipe in place above the ground in places where the pipe was not touching the ground. The side-slopes of the dam may not be consistent from the top to the bottom causing the pipe to be above the ground in areas. The recommended brace consists of a 10 cm by 10 cm (4 in by 4 in) post placed on both sides of the pipe in the ground with a 1 m (3 ft) deep hole with concrete poured in to secure the posts. Braces that hold the pipe to the ground have one 5 cm by 15 cm (2 in by 6 in) board cut to length connecting the two posts, placed touching the top of the pipe. Braces that hold the pipe above ground, if required, should have two 5 cm by 15 cm (2 in by 6 in) boards, one connecting the posts above the pipe and one connecting the posts below the pipe, both in contact with the pipe. Having these braces secure the pipe is crucial as discharge events put strain and pressure on the pipe, which could damage the pipe or joints if allowed to move.

Regular maintenance includes keeping the spillway pipe painted and keeping it properly secured and braced. The best paint color for longevity of the pipe is Carbon Black (Ohio DNR, 2009). It would be best to paint the pipe after installation to keep the paint from interfering at the joints where the sections are connected to one another.

DISCUSSION

Case Study in Design of Siphon Spillway

A demonstration site was established on LaMaster Dairy Farm in Clemson, South Carolina. ArcGIS-ESRI (Redland, California) was used to delineate the pond. A 1.73 ha (4.27 ac) pond was constructed in the 1960's for crop irrigation and livestock watering. The primary or vertical riser spillway had deteriorated and lowered the normal pool of the pond several feet. Reduced functionality of the water control structure caused concern for the farm manager. This produced a great opportunity for a pond rehabilitation project.

There were no plans or drawings of the pond from the original time of construction, so a hydrologic study was needed. ArcGIS was utilized to delineate the watershed area using USGS topographic maps and ArcGIS was used to give an accurate depiction of land use and cover. It was determined that the watershed area was 47.8 ha (118 ac). Other information found using the topography maps was the watershed length and slope. The watershed length was 1800 feet and the average slope of the watershed was 4.4 percent. The weighted CN was obtained by delineating land use in the watershed and using the procedure shown in Table 3.2.

Table 3.2. LaMaster Weighted CN Table. Shows calculations of weighted CN for LaMaster Watershed

Land Use	Hydrologic Group	Area (ac)	CN	A x CN
Forest	Group B	60	55	3300
Water	Group B	4	0	0
Pasture	Group B	52	61	3172
Parking Lot/Roof	Group B	2	98	196
Sum		118		6668
Weighted CN	56.5			

Using EFH 2, an engineering tool provided by NRCS (EFH-2, 2009), the runoff was estimated. EFH-2 uses is a runoff calculator that uses the TR-55 method. This tool calculates the watershed runoff data after the user inserts the previously calculated drainage area, curve number, watershed length, and watershed slope. A 25-year, 24-hr storm event was used to design the emergency spillway. Table 3.3 shows the results when considering different rainfall events.

Table 3.3. LaMaster Hydrologic Study Results. Table Shows the results from the hydrologic study for the LaMaster pond watershed.

	Frequency (years)	24 Hour Rain (in)	Peak Flow (cfs)	Runoff (in)
Storm 1	1	2.99	8	0.24
Storm 2	2	3.58	19	0.45
Storm 3	5	4.45	43	0.83
Storm 4	10	5.24	71	1.23
Storm 5	25	6.52	124	2
Storm 6	50	7.72	178	2.8
Storm 7	100	9.17	249	3.86

The emergency spillway was designed by using Equation 3.6 and inserting the flow rate for the 25 year return period rainfall event. A 25 year return period rainfall event was the minimum requirement dictated in the SC NRCS Pond Standard (2011). The minimum width for the emergency spillway was calculated to be 11.8 m (38.8 ft). Conservatively, a 19.8 m (65.0 ft) wide emergency spillway was installed. The layout of the area allowed for a wider emergency spillway than that required. This allows for a higher flow rate and a lower velocity for a given flow rate causing less erosion and therefore requiring less maintenance.

The siphon spillway was designed by calculating the storage in the pond. The pond was measured on ArcGIS at 4.3 acres and has a two foot elevation difference between normal pool and emergency spillway. The pond has 8.5 acre-feet or 372,000 ft^3 of flood storage. Discharging that amount of water over ten days requires 0.4 cfs (193 gpm). This flow rate could be achieved by installing a 150 mm (6 in) siphon, but 200 mm (8 in) PVC pipe was used for this project. Using a larger pipe diameter for the siphon spillway reduces the amount of time needed to discharge flood storage.

Case Study in Installation of Siphon Spillway

A temporary 6.4 cm (2.5 in) diameter siphon installed above the dam was used to lower the water level of the pond on this project. The pond only needed to be drained a few inches to install the siphon on this project since the primary spillway had deteriorated and lowered the pond level. The temporary siphon was engaged for 4 days and lowered the pond 0.24 ha m (2 ac ft). The temporary siphon had a valve on the inlet and outlet, which held the water until the siphon had been manually filled completely. Once it had been filled manually, the inlet and outlet valve were opened at the same time to create a vacuum causing drainage of the pond until the inlet was above the water. Once air is introduced into the system, the siphon no longer had suction and no longer discharged.

Figure 3.6. Temporary siphon figure. The temporary siphon that was used to lower water level of pond before construction of LaMaster Siphon.

During excavation, surveying equipment was used to obtain designed elevation for placement of the barrel pipe through the dam. This included one person running the surveying level and notebook, one person using the surveying rod, and one person operating the backhoe. Having multiple people on site during construction helped the project run smoothly.

After excavation, the pipe sections were cut to fit and joined by 45-degree elbows on both sides of the dam. The elbows led from the barrel down to the outlet and the inlet sections of the spillway. A 45-degree joint was glued to the inlet at a side angle of approximately 10° to keep the inlet from being positioned on the bottom of the pond. This kept the inlet protected from clogging. Once the inlet and outlet had been installed, the cross section of the dam that was removed was replaced.

An end cap with drilled holes throughout a section of pipe was used as the inlet guard for the LaMaster Siphon, as shown in Figure 3.2. The pipe had 325 cm^2 (50.3 in^2) of opening for water to travel through. When putting a cap on the end and drilling openings in the pipe, it is recommended to give four times the amount of area to reduce resistance to flow and to safeguard against clogging (Georgia NRCS, 2012). This system required 260 holes that measured 2.5 cm (1 in) in diameter to be drilled into inlet after it was capped.

A guard composed of four 0.64 cm (0.25 in) diameter lengths of threaded rod put in parallel to one another was used at the outlet (Figure 3.8). This was done to reduce the amount of resistance in the outlet compared to the type of inlet guard used, while still protecting the spillway from animals that could disrupt functionality.

After the pipe was installed, braces were put in every 2 to 3 m (5 to 10 ft) to secure the spillway (Figure 3.9). These keep the spillway secure during high discharge events. The braces consisted of 10 cm by 10 cm (4 in by 4 in) posts put 0.9 m (3 ft) into the ground, with concrete to secure the posts. The posts were connected by 5 by 15 cm (2 in by 6 in) posts cut to length to join the posts on both sides of the spillway.

Figure 3.7. Outlet guard figure. Shows outlet guard used during install of siphon spillway.

Figure 3.8. Brace Figure. The left side of this figure shows a brace securing the pipe above ground. The right side of this figure shows a brace placed to secure the pipe to the ground.

Once braces were installed, the last step was to install the vent. The vent was installed as shown in Figure 3.1 and cut at the designed normal pool elevation. After the vent is installed as designed, the siphon spillway will be ready to replace the hydraulic function of the dilapidated vertical riser spillway.

The last step in this project will be to stop the hydraulic function of the vertical riser spillway. It was recommended by SC NRCS to mix 23 kg (50 lbs) of bentonite clay per 0.8 m^3 (1 yd^3) of concrete required. The bentonite clay swells and improves sealing the vertical riser spillway. At the LaMaster site, 3 yd^3 of concrete were required for the vertical riser sealing along with 68 kg (150 lbs) of bentonite clay. After concrete is poured, no water can be discharged through the vertical riser. A metal guard will be installed over the top of the vertical riser for further protection to recreational users of the pond.

Case Study Budget Analysis

A budget analysis was performed for the LaMaster Siphon to evaluate how cost effective the siphon system was versus installing a vertical riser system. All material costs were accurate as of Spring 2014. A traditional riser system was designed to get an accurate cost estimate of materials needed for this alternative. The estimate and list of materials is included as Appendix B. A survey was conducted to assist in determining the amount of earthwork required. The earthwork required for the vertical riser was estimated by using a dam depth of 7.3 m (24 ft), dam width of 3.7 m (12 ft), and working width (perpendicular to dam) of 0.9 m (3 ft). The slopes of the excavation for installation of the pipe were designed at 2 to 1 slopes for a stability. The front and back slope of the dam were figured to be 2 to 1 slopes. Equation 3.7

was used to estimate the yardage of earthwork required to reinstall a vertical riser spillway on the LaMaster pond The equation approximated the excavation as the sum of three volumes: a trapezoidal prism extending down from the top of the dam across its top width, and two trapezoidal based pyramids, one at the front slope and one at the back slope.

$$E=(2D^2+3D)T+\left(\frac{Z_b+Z_f}{3}\right)(2D^3+3D^2) \qquad \text{(Eq. 3.7)}$$

Where E = earthwork required, yd^3

D = depth of dam, ft

T = top width of dam, ft

Z_b = back slope of dam, X value in slope ratio

Z_f = front slope of dam, X value in slope ratio

Application of Eq. 3.7 for the LaMaster Pond resulted in an estimated cut volume required of 1,995 yd^3

For the siphon spillway, pipe cost was estimated using 30.5 m (100 ft) of 200 mm (8 in) Schedule 40 PVC, four 200 mm (8 in) couplings, six 200 mm (8 in) (45°) elbows, 1 200 mm (8 in) branch tee, an adapter reducing from 200 mm (8 in) to 50 mm (2 in) ,two 50mm (2 in) 45° elbows, and 6.1 m (20 ft) of 50 mm (2 in) schedule 40 PVC pipe. Labor cost was estimated by interviewinga local contractor (Arvid Aartuun, Table Rock Realty, November 2013). Earthwork was accounted for by using a trench calculation as shown in Equation 3.8. This was developed by using the same method of installation that was used in the LaMaster siphon case study. The trench removed to install the siphon at LaMaster was vertical and had no side slopes.

$$V_c = \frac{DWL+2D^2}{27} \qquad \text{(Eq 3.8)}$$

Where V_c = cut volume for earthwork required, yd^3

D = depth of trench,

W = width of the trench, ft

L = length of the trench, ft

Table 3.4. Budget Analysis

	Budget for Riser System			
Earthwork	1995 yds^3	7.50	per $yard^3$	$ 14,962.50
Pipe Expense				$ 4,803.90
Labor for Pipe Installation	60 hours	12.50	per hour	$ 750.00
Seeding(lime, mulch, grass)				$ 600.00
Riser System Total Cost				$21,116.40
	Budget for Siphon			
Earthwork	6.5 yds^3	7.5	per $yard^3$	$ 48.75
Pipe Expense				$ 1,600.00
Labor for Installation	40 hours	12.50	per hour	$ 500.00
Seeding (lime, mulch, grass)				$ 200.00
Siphon System Total Cost	$ 2,348.75			
Saving from using Siphon	$ 18,767.65			
Percent Savings by Using Siphon	89%			

CONCLUSION

A set of design and construction procedures were evaluated and demonstrated through the installation of a siphon spillway on LaMaster Dairy Farm in Clemson, South Carolina. Equations were shown to calculate runoff and a program was cited that is freely available that assists in calculating runoff. Costs incurred on the LaMaster Dairy Siphon project were used with general material costs in order to compare and evaluate siphon spillway versus replacement of vertical riser system for spillway rehabilitation. The cost analysis demonstrated that a siphon spillway is economical when compared to installing a traditional riser system.

Design assistance for siphon spillways can be obtained for no cost at NRCS Service Centers. This is a part of their technical assistance that they are to provide to the public.

Future work and observations could be done to quantify maintenance of the siphon spillways. PVC use in this function is relatively new compared to the other materials and there will be unforeseen challenges associated with it in the future.

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CHAPTER 4

THESIS CONCLUSIONS

Siphon spillways are a viable alternative during pond rehabilitation. The reduced earthwork along with the lengthy expected lifespan of materials when compared to other water control structures combine to make siphon spillways an economical alternative for pond rehabilitation as shown in the budget analysis in Chapter 3. This compilation of information on siphon spillways should serve as a useful guide to landowners interested structures.

Chapter 2 explored siphon performance by way of flow rate and minimum required vent opening. Flow rate was explored to provide better sizing information for design of siphon spillways. Vent sizing was explored due to noted instances of ponds continuing to drain until empty with use of siphon spillway. A flow prediction equation was developed but requires further validation. A validated minimum required vent diameter model was developed and should be confirmed on other siphon spillway systems.

The design and construction guidelines in Chapter 3 provide concise instructions and can be used as a handbook for installing siphon spillway systems. Information on these topics will be useful to many people as pond rehabilitation projects increase as structures age. The budget analysis in Chapter 3 shows the economics of siphon spillways with a real world example of a pond rehabilitation project. This information on siphon spillways as related to pond rehabilitation has the power to influence management decisions for landowners.

Appendix A: Runoff Curve Numbers (Iowa NRCS, 2008)

Cover description		***Curve numbers for hydrologic soil group***			
Cover type and hydrologic condition	**Average impervious area[2]**	**A**	**B**	**C**	**D**
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc)[3]:					
Poor condition (grass cover <50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover >75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc (excluding ROW)		98	98	98	98
Streets and roads:					
Paved: curbs and storm sewers (excluding ROW)		98	98	98	98
Paved: open ditches (including ROW)		83	89	92	93
Gravel (including ROW)		76	85	89	91
Dirt (including ROW)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only)[4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1-2 inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation)[5]	77	86	91	94	
Idle lands (CN's are determined using cover types similar to those in Table 3)					

1 Average runoff condition and I_a=0.2S.

2 The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions pay be computed using Figures 3 or 4.

3 CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.

4 Composite CN's for natural desert landscaping should be computed using Figures 3 or 4, based on the impervious area percentage (CN=98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.

5 Composite CN's to use for the design of temporary measures during grading and construction should be computed using Figures 3 or 4 based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

Cover description			*Curve numbers for hydrologic soil group*			
Cover type	**Treatment**[2]	**Hydrologic condition**[3]	**A**	**B**	**C**	**D**
Fallow	Bare soil	--	77	86	91	94
	Crop residue cover (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR+CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C+CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured and terraced (C&T)	Poor	66	74	80	82
		Good	62	71	78	81
	C&T+CR	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR+CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C+CR	Poor	62	73	81	84
		Good	60	72	80	83
	C&T	Poor	61	72	79	82
		Good	59	70	78	81
	C&T+CR	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded or broadcast legumes or rotation meadow	SR	Poor	66	77	85	89
		Good	58	72	81	85
	C	Poor	64	75	83	85
		Good	55	69	78	83
	C&T	Poor	63	73	80	83
		Good	51	67	76	80

1 Average runoff condition and I_a=0.2S.
2 Crop residue cover applies only if residue is on at least 5% of the surface throughout the year.
3 Hydraulic condition is based on combination factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or close-seeded legumes, (d) percent of residue cover on the land surface (good ≥20%), and (e) degree of surface roughness.
Poor: factors impair infiltration and tend to increase runoff.
Good: factors encourage average and better than average infiltration and tend to decrease runoff.